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EXPERIMENTAL EVALUATION ON SEISMIC PERFORMANCE OF SANDWICH BOX COLUMNS

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SUMMARY

This paper presents experimental information on the behaviour of sandwich box columns subjected to combined bending and axial loading. The sandwich box columns consisted of double thin-walled steel tubes with concrete between them. Owing to the interaction between steel and concrete, the composite members performed in a ductile manner during testing. Test results show that high strength/mass ratio characteristics were maintained, and the goal of obtaining significant member ductility was also achieved. The contribution of concrete to member performance was found to be more significant for members with higher steel width/thickness ratios. Copyright © 1999 John Wiley & Sons, Ltd.

KEY WORDS: cyclic behaviour; sandwich box column; local buckling; strength; ductility

1. INTRODUCTION

Hollow steel and reinforced concrete box piers are commonly utilized in bridge construction. Strength/mass ratios higher than those of solid sections make the hollow-section designs more competitive than solid ones, because they provide sufficient strength, while simultaneously reducing the weight of the system.

Steel is stronger than other construction materials such as timber and concrete; therefore, steel box piers can be constructed with smaller member dimensions yet still satisfy the same loading requirement. However, such a trade-off lowers the lateral stiffness of steel constructions and allows greater lateral displacements, thus reducing system service levels. Furthermore, the hollow-section walls are susceptible to local buckling if width/thickness ratios are not maintained.

Hollow reinforced concrete piers are generally considered to be economical designs that provide high compressive strength. However, such designs lack ductility and are short on shear strength when application in seismic zones is considered. Specification[1] requires that pier stirrups used in areas with high seismic activity be fabricated with close spacing so that the members can

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perform in a ductile manner. However, this method entails difficulties in engineering practice because effective fabrication of reinforced concrete is not easy to achieve within the limited wall-thickness spaces.

Composite steel and concrete forms have been studied for years,[2–5] and have been proven to be effective with respect to strength and ductility. Such performance is achieved through interaction between the steel and concrete. Structural steel–concrete composites are commonly seen in concrete-filled tubes (CFT) used in urban bridge construction. Recently published studies[6] show that the compressive strength of a CFT member's concrete core increases with better confinement by the steel tube, and the occurrence of local buckling of the steel tube is delayed until large lateral displacements are imposed, which represents better ductility performance. In engineering practice, steel tubes also serve as molds during construction and perform structural functions after construction is finished; therefore, they also provide the further benefit of lowering construction costs.

The advantages of using CFT members are described above. However, CFT members used in bridges with high elevations, and in bridges with long spans and heavy loads, must be very large, and their weight limits their applications in engineering practice. Therefore, studying modified structural forms to improve the performance of such members is essential.

A recent study[7] showed that the compressive strength of a composite tubular member made of double thin-walled steel tubes and concrete was higher than the sum of the strengths of the individual components. The concrete–steel composite worked well for members under compression; however, in order to use such members in high seismic-activity areas, their behaviour under combined axial and cyclic lateral loading needs further investigation because concrete is vulnerable to tensile forces induced by earthquakes. Therefore, the authors of the present study developed an experimental program to study the behaviour of hollow sandwich box columns under the above-mentioned loading. Strength performance and ductility of such members was studied in order to establish fundamental design rules for application in high seismic-activity zones.

2. RESEARCH OBJECTIVES

The main objective of this study was to provide fundamental information on the behaviour of sandwich box columns under cyclic loading. Detailed objectives include: (1) conduct an experimental investigation of sandwich box members subjected to combined axial and lateral loading to gather empirical information; (2) evaluate the performance of those members to verify the validity of such designs; and (3) provide simplified expressions of member properties for engineering practice, where possible.

3. EXPERIMENTAL PROGRAM

3.1. Specimens

Twenty-one specimens, including three CFT and 18 sandwich members, were fabricated for testing. Cross-sections for the CFT and sandwich members are shown in Figure 1. Height of all specimens was 900 mm. Sandwich specimens consisted of double thin-walled tubes with concrete

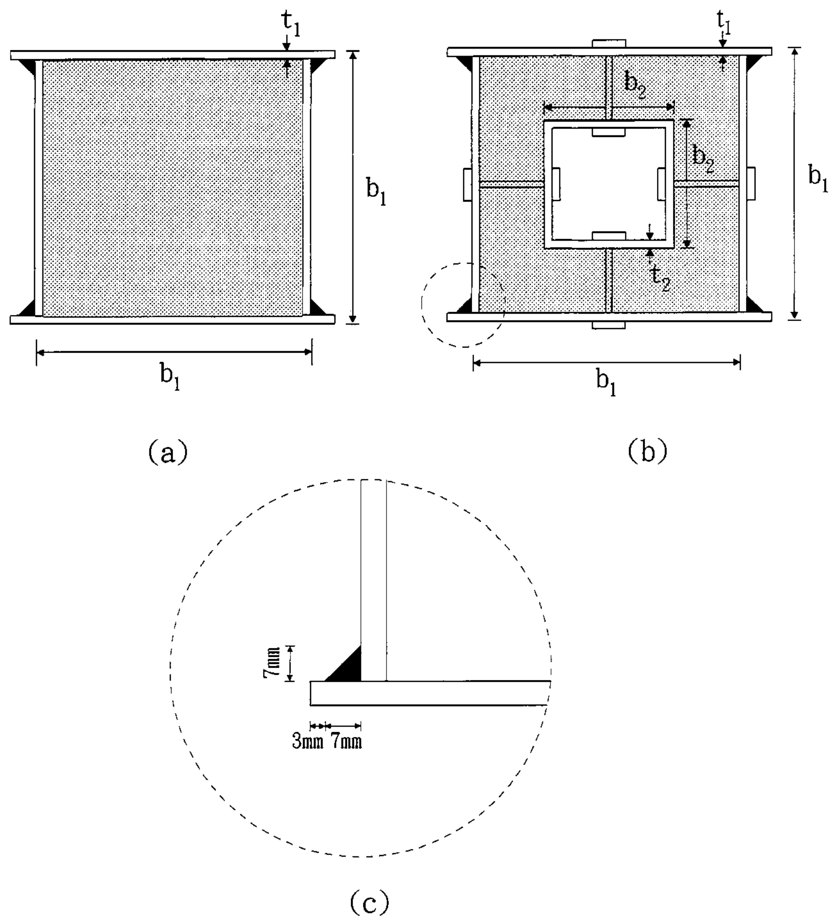

Figure 1. Description of test specimens: (a) cross-section for CFT; (b) cross-section for sandwich members; (c) details of corner fillet welds

between them. Two types of inner tube were used, JIS SS41-grade tubes with cross-sectional dimensions ($b_2 \times b_2 \times t_2$) of $100 \times 100 \times 4{\cdot}5$ and $150 \times 150 \times 6$ mm, respectively. Therefore, the width/thickness ratios for the two inner tubes were 22·2 and 25, respectively. Yield stresses of the smaller and larger inner tubes were 330 and 313 MPa, respectively. Outer tubes were fabricated by fillet-welding four JIS SS41 thin plates of different thickness (t_1): 4·5, 6, and 7 mm, so that the width/thickness ratios of the plates could be adjusted to study variations in behaviour between members with compact and non-compact plates. Yield stresses of the steel plates used in making the outer tubes were 240, 246, and 228 MPa, respectively, for plate thickness equalling to 4·5, 6·0 and 7·0 mm.

Test specimens were divided into three groups according to the outer tube thickness. Nominal width of all test specimens (b_1) was 280 mm; therefore, width/thickness ratios of the outer tubes for the three groups of specimens with outer tube thickness equalling to 4·5, 6, 7 mm, referred to as the T5, T6, and T7 test series, were 62·22, 46·67, and 40, respectively, which represented

non-compact and compact plate elements as specified in design code.[8] Ordinary construction-grade concrete was used to fill the gap between the double tubes. Concrete strength was determined using the cylinder compressive test after a 28-day curing process, and was found to be 20·6 MPa on average.

3.2. *Specimen fabrication*

Specimens were made by welding inner and outer tubes to $500 \times 500 \times 30$ mm plates on the top and bottom. Eight bolt holes were drilled in the bottom plates so that the specimens could be bolted to a strong test platform, and six holes were drilled in the top plates to attach the specimens to the stiffened loading beam. Tie rods were used to link the inner and outer tubes forming composite structures and to study whether shear-connector-type devices are helpful in bonding the concrete and steel tubes. Each group of sandwich specimens was divided into three sub-groups: members with one tie rod at mid-height, tie rods at quarter-heights, and members with no ties at all.

Placement of tie rods was completed by first drilling 13-mm-diameter holes in both tubes and then snugly fitting the rods. Specimens were finished after the concrete was poured. The smallest ratio between concrete thickness and cross-sectional dimension was 0·2. It was observed during the fabrication process that concrete could be easily poured into specimens with very small gaps between the double tubes, and into specimens with maximum numbers of tie rods, thus demonstrating the ease of fabrication and justifying the applicability of sandwich members to engineering practice.

3.3. *Specimen identification system*

Specimens were categorized according to outer-tube thickness, inner-tube type, and numbers of tie rods used. Each group of specimens contained one CFT member, three sandwich members with smaller inner tubes, and three sandwich members with larger inner tubes. A plain concrete-filled tube with tube thickness equalling x mm was designated as Tx only; however, corresponding sandwich specimens with x-mm-thick outer tubes, smaller/larger inner tubes (labelled S and B respectively) and number of tie rods equalling y were designated as TxSy and TxBy, respectively. For example, a T6B1 symbol represents a specimen consisting of a larger inner tube ($150 \times 150 \times 4$ mm), a 6-mm-thick outer tube, and a single tie rod at mid-height. Following the same notation system, T6S3 indicates a specimen with a 6-mm-thick outer tube, a smaller inner tube and three tie rods, one at each quarter-height point. A plain concrete-filled tube 6 mm thick is designated as T6.

3.4. *Test set-up*

The test set-up, as shown in Figure 2, consisted of one actuator, one hydraulic jack, one loading beam for load transmission, and one base platform. Test specimens were first bolted to a stiffened base platform rigidly fastened to the strong floor. The stiffened loading beam was placed atop the specimens linking them to a servo-controlled hydraulic actuator. The actuator used for this test was a SCHENCK PGz 1·0x with maximum capacity of 1000 kN. One end of the actuator was fastened to the reaction wall and a swivel on the other end was bolted to the loading beam using ASTM A325 high-strength bolts. Axial loading on specimens was generated by a double-action

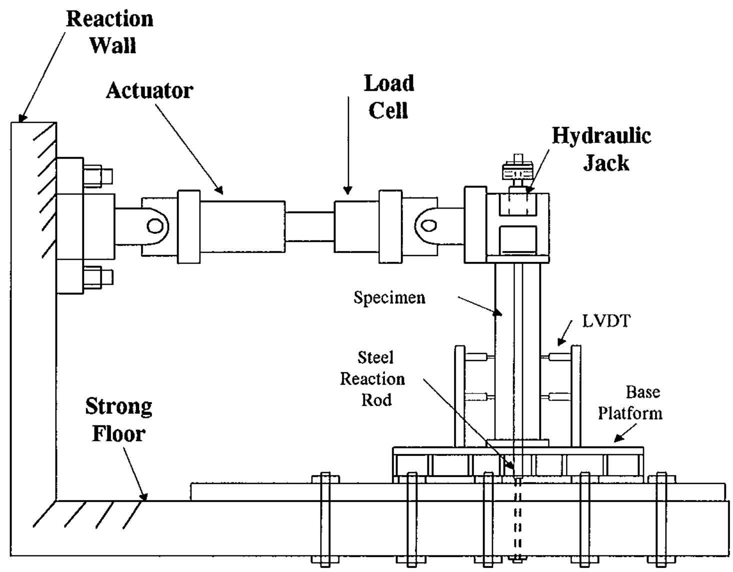

Figure 2. Test set-up

hydraulic jack pushing against a stiffened reaction beam supported by a pair of high-strength reaction rods hinged where they were attached to the strong floor. Test data was stored in an Intel 586 personal computer for later analysis.

3.5. Test procedures

Test specimens were subjected to constant axial loading and cyclic lateral forces. Axial loading magnitude was set to 10 per cent of specimens' compressive yield strengths to take into account the effect of superstructure weight. Lateral forces were generated by a set of prescribed increasing cyclic displacements of the actuator atop the specimens. A typical displacement history is shown in Figure 3. This test procedure was designed to evaluate the ultimate capacities, rather than the time-dependent responses, of sandwich members under lateral forces so that design references could be established. Applied lateral forces and resulting displacements were measured using load cell and internally mounted Linear Variable Differential Transformer (LVDT). Member responses were first measured by strain gauges whose outputs were amplified by a signal conditioner, and then sent to the personal computer through the data-acquisition system.

4. GENERAL BEHAVIOUR

When test specimens were subjected to combined axial and lateral loading, yielding was first observed at the bottoms of the specimens where maximum stresses occurred. However, initial local buckling of the outer tubes occurred at distances equal to one-half the widths of the specimens as measured from their bottoms. The in-filled concrete crumbled when the steel tubes

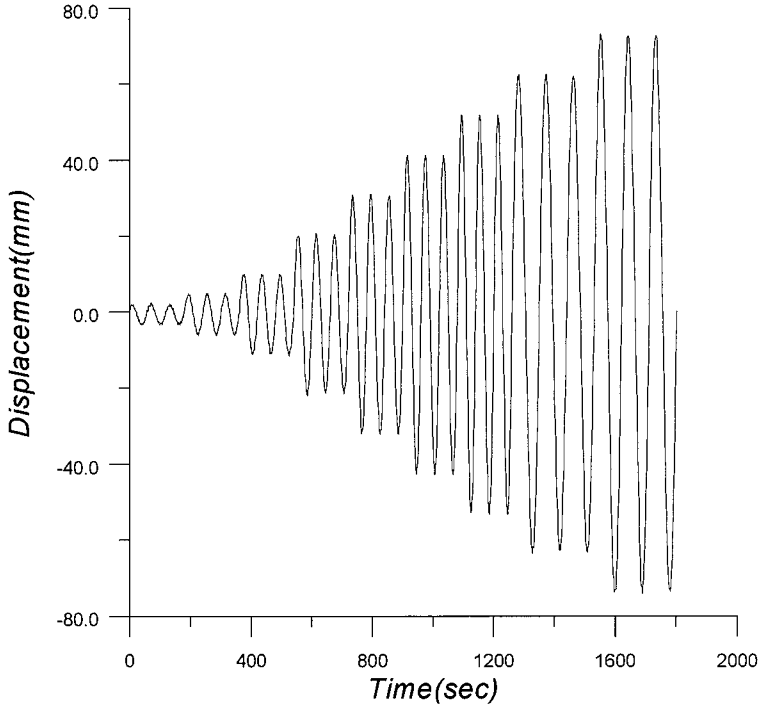

Figure 3. Typical displacement history (for T5 test series)

buckled; however, due to support from the in-filled concrete, specimens did not show significant deterioration in strength until the buckled plates fractured at the corners of flange plates allowing the crushed concrete to leak out in powdery form. At this stage, the strength of the specimens suddenly dropped, and deterioration accelerated until specimens showed obvious damage. These results showed that the in-filled concrete, even after crushing, effectively maintained the stability of the steel tubes, thus emphasizing the significant ductility performance of concrete-filled sandwich members. Hysteresis curves for the test specimens are shown in Figure 4 and failure modes of sandwich members are shown in Figure 5.

5. INTERPRETATIONS OF TEST RESULTS

5.1. Strength and allowable drift

T5 test specimens exhibited local buckling of steel tubes at a lateral displacement of 10·4 mm which is equal to a 1·16 per cent lateral drift. However, sandwich box members in the T5 series sustained drifts of 2·32 per cent before initiation of local buckling. After initial buckling occurred, buckled plates were repeatedly bent and straightened during cyclic displacement and their strength continued to grow due to the contribution of concrete in resisting deformation of the steel plates. Maximum strengths for all specimens in the T5 series were reached at drifts equalling

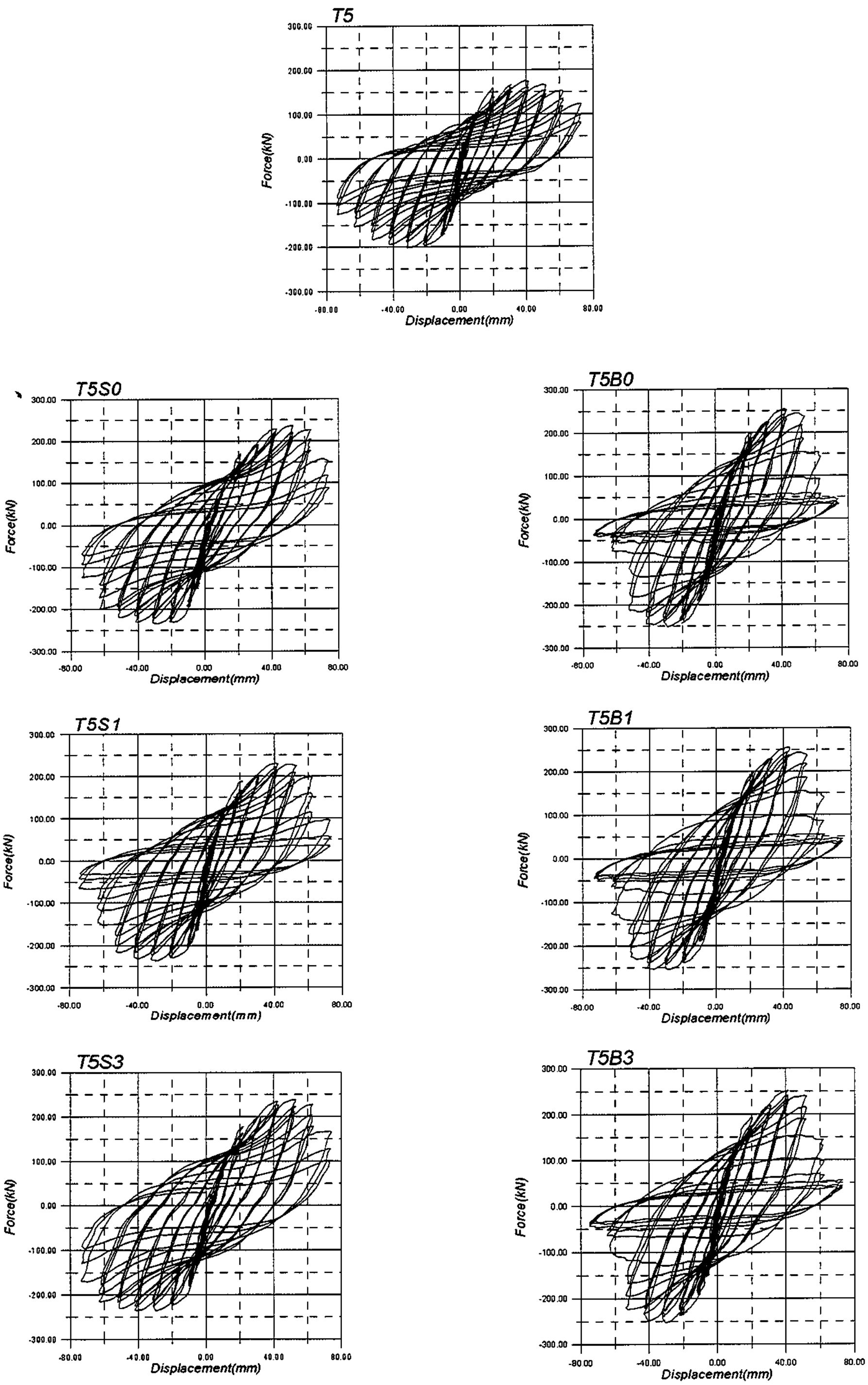

Figure 4(a). Hysteresis curves: T5 test series

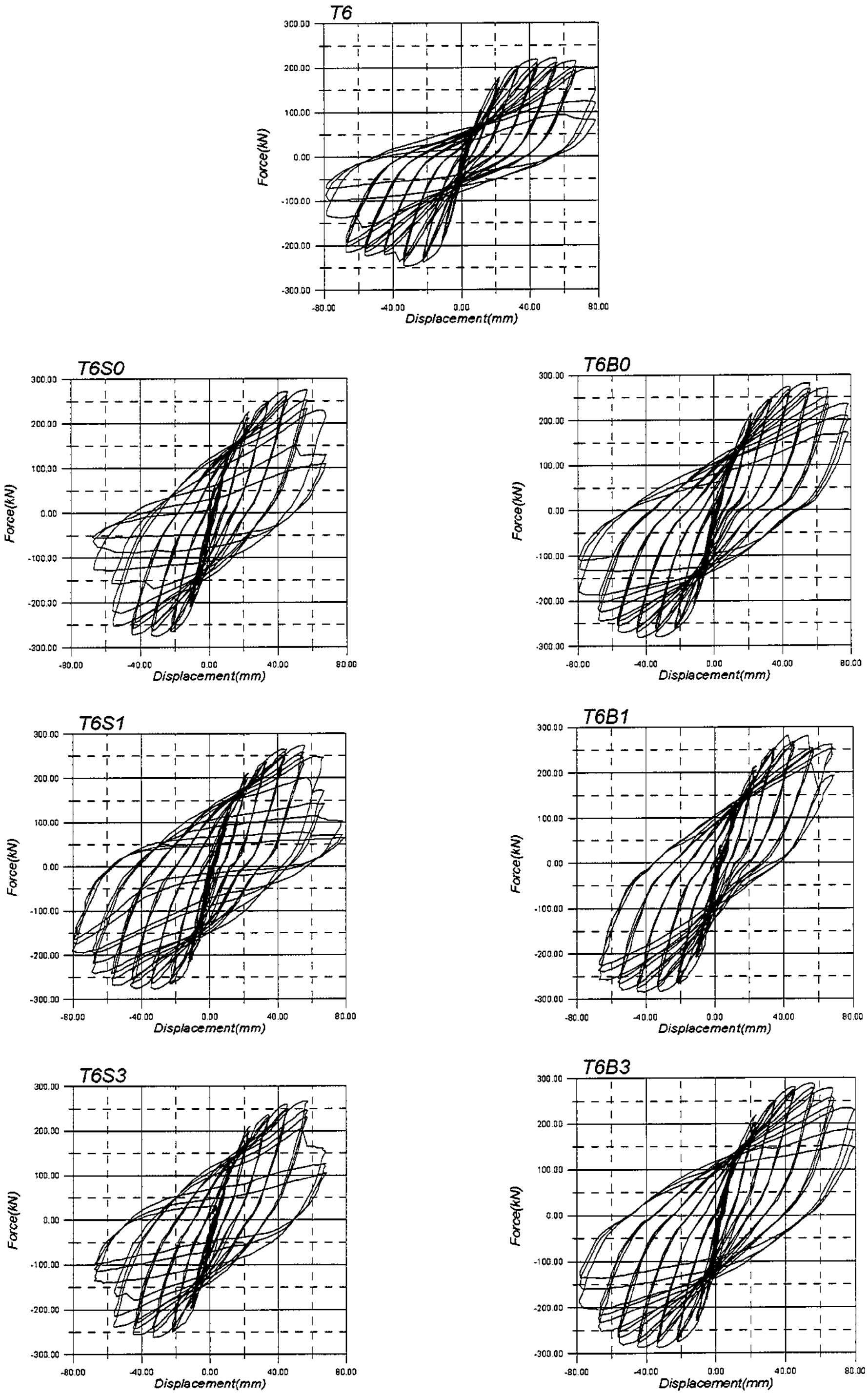

Figure 4(b). Hysteresis curves: T6 test series

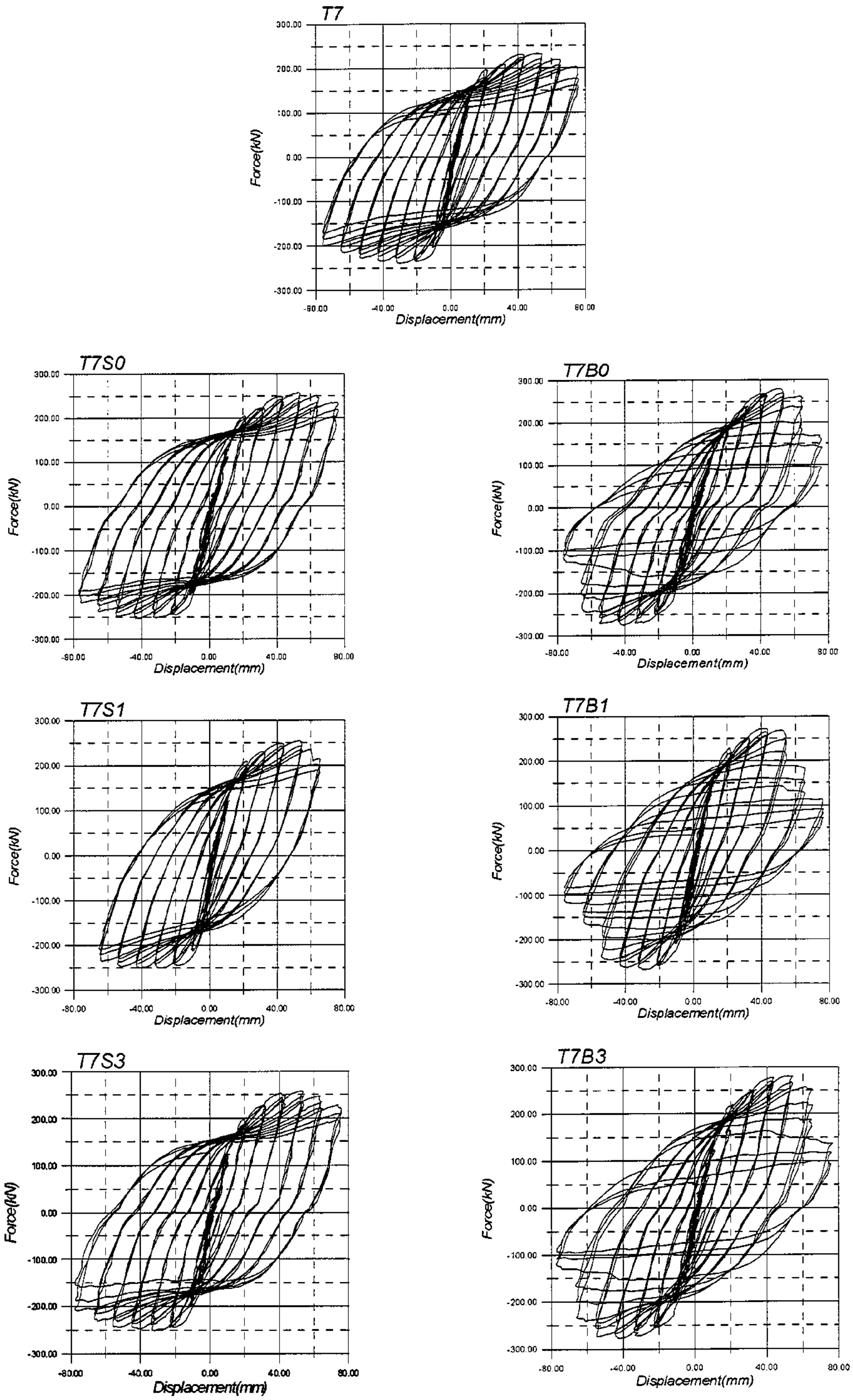

Figure 4(c). Hysteresis curves: T7 test series

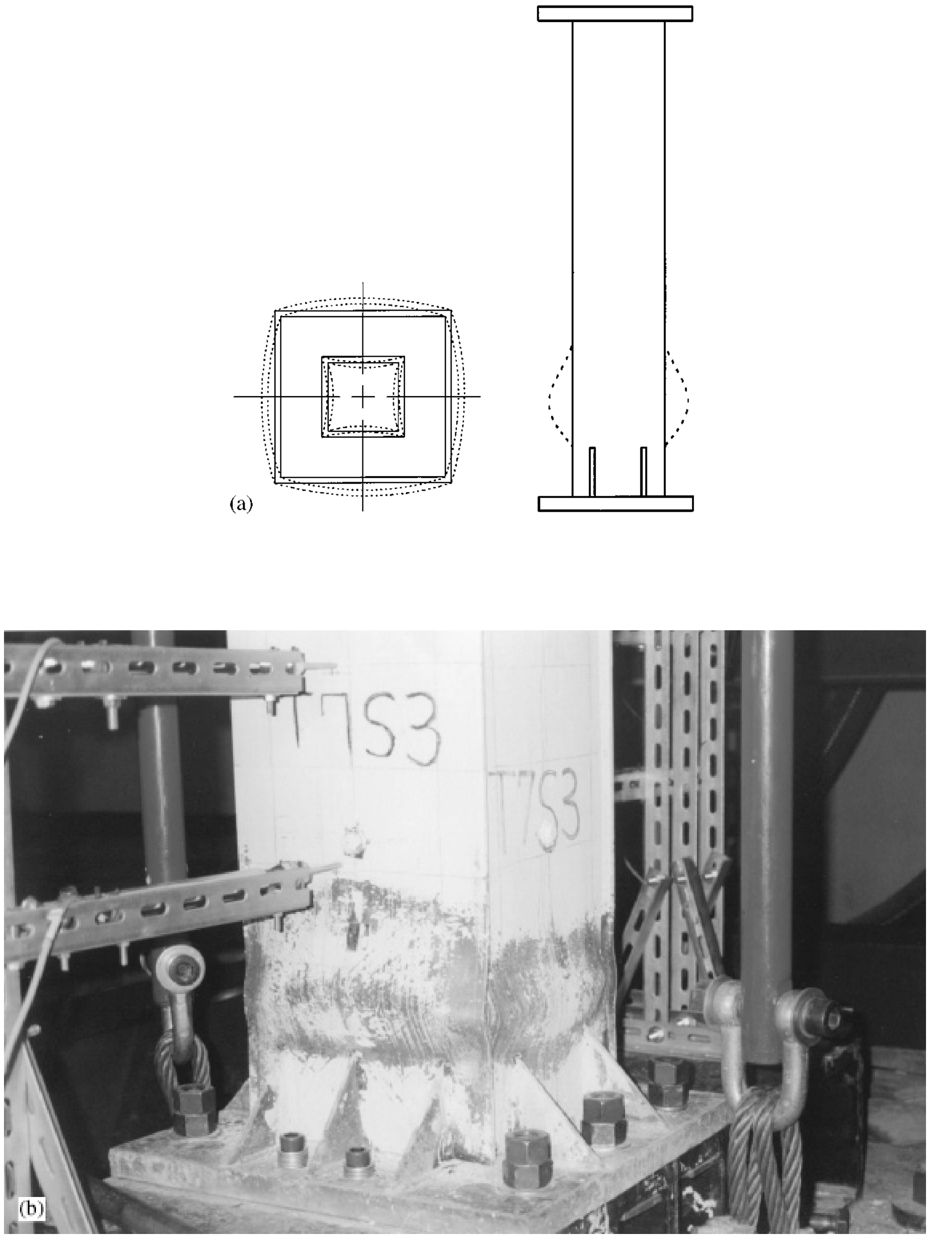

Figure 5. Failure mode of sandwich members: (a) buckled shape, and (b) photograph of test specimen

4·62 per cent. Compared with that of T5 CFT specimen, a 45 per cent strength enhancement was achieved in the sandwich member with larger inner tube (T5B0). The sandwich member with a smaller inner tube (T5S0) displayed lower strength gain than T5B0; however, it was still 35 per cent stronger than T5.

The drift ratio at which local buckling occurred in all T6-series specimens was 3·73 per cent. Maximum strength was reached when drift equalled 6·2 per cent. Normalized strengths of sandwich members with larger and smaller inner tubes were 1·31 and 1·25, respectively, with respect to the T6 CFT specimen. Local buckling of T7 series outer tubes occurred when lateral drift reached 3·6 per cent, and maximum strength was reached when drift equalled 6 per cent. The allowable drift performance for T6 and T7 series specimens were similar because they both had outer tubes with compact elements. Normalized strength with respect to that of the T7 CFT specimen for sandwich members in T7 test series with larger and smaller inner tubes were 1·20 and 1·10, respectively. Normalized strengths for all sandwich members with respect to their corresponding CFT members are listed in Table I.

By comparing the normalized strength among these three series, a preliminary conclusion can be drawn that strength gains increased as outer-tube width/thickness ratios increased, particularly for those specimens with non-compact plates, because concrete delays the buckling of non-compact plates. In addition to the strength enhancements, effectiveness of sandwich members was also validated by their significant deformation capacities described above.

5.2. *Strength/mass ratio*

In order to compare structural form efficiencies, test specimen strength/mass ratios were further examined. The highest normalized strength/mass ratios with respect to that of T5 CFT specimen for sandwich members in the T5 series with larger and smaller inner tubes were 1·73 and 1·43, respectively. Normalized strength/mass ratios of sandwich members with respect to that of solid CFT member in the T6 series were slightly lower than those in the T5 series, at 1·51 and 1·31, respectively. The strength/mass ratios of specimens in the T7 series were 1·42 and 1·16 for sandwich members with larger and smaller inner tubes, with respect to solid one.

Table I shows that normalized strength/mass values are closely related to ratios of concrete areas (A_c) to steel areas (A_s). When A_c/A_s ratios are increased, normalized member performances decrease accordingly. The relationship between A_c/A_s ratios and normalized strength/mass ratios for all specimens are shown in Figure 6.

5.3. *Effect of tie rods*

A comparison of member strengths for the three test series is plotted in Figure 7. It can be seen that the behaviour of members with the same inner and outer tubes, regardless of the number of tie rods, was similar up to the maximum strength point and only minor differences were observed after the plate corners fractured due to repeated heavy stress concentrations. This phenomenon revealed that the bonding between concrete and the double steel tubes was effectively achieved in the tested sandwich members; therefore, the shear connectors seemed to be unnecessary. However, for real structures, the bonding strength needs to be very large to assure that sandwich members function as composites, and this is not easy to achieve. Therefore, the tie rods may be very important and helpful in the design of realistic sandwich structures.

Careful examination of the test specimens was conducted after the tests. It was found that failure due to buckling was only located at a height equal to one-half of the width as measured from the specimen bottoms, and that this was smaller than the distances between tie points. By examining the failure modes of the inner and outer tubes as shown in Figure 5, it can be seen that both tubes moved outward with respect to the concrete infills. Had tie rods been placed where

Table I. Normalized strength/mass ratios and α values

Specimen	A_s (mm^2)	A_c (mm^2)	A_c/A_s	Unit weight (kN/mm)	Strength (kN m)	Normalized strength	Normalized strength/mass ratio	α
(a) *T5 specimens*								
T5	4959·0	73441·0	14·81	0·00205	197·529	1	1	0·345
T5S0	6495·0	63441·0	9·77	0·00194	266·499	1·35	1·43	0·745
T5B0	7295·0	50941·0	6·98	0·00172	286·362	1·45	1·73	0·425
T5S1	6495·0	63441·0	9·77	0·00194	260·430	1·32	1·39	0·605
T5B1	7295·0	50941·0	6·98	0·00172	288·017	1·46	1·74	0·400
T5S3	6495·0	63441·0	9·77	0·00194	269·257	1·36	1·46	0·495
T5B3	7295·0	50941·0	6·98	0·00172	283·604	1·44	1·71	0·380
(b) *T6 specimens*								
T6	6576·0	71824·0	10·92	0·00214	249·394	1	1	0·800
T6S0	8112·0	61824·0	7·62	0·00203	310·639	1·25	1·31	0·795
T6B0	8912·0	49324·0	5·53	0·00181	318·364	1·28	1·51	0·900
T6S1	8112·0	61824·0	7·62	0·00203	308·984	1·24	1·31	0·360
T6B1	8912·0	49324·0	5·53	0·00181	318·364	1·28	1·51	0·395
T6S3	8112·0	61824·0	7·62	0·00203	301·811	1·21	1·28	0·370
T6B3	8912·0	49324·0	5·53	0·00181	325·537	1·31	1·54	0·430
(c) *T7 specimens*								
T7	7644·0	70756·0	9·26	0·00220	262·636	1	1	0·610
T7S0	9180·0	60756·0	6·62	0·00209	289·673	1·10	1·16	0·360
T7B0	9980·0	48256·0	4·84	0·00187	316·157	1·20	1·42	0·370
T7S1	9180·0	60756·0	6·62	0·00209	287·466	1·09	1·15	0·605
T7B1	9980·0	48256·0	4·84	0·00187	306·776	1·17	1·37	0·590
T7S3	9180·0	60756·0	6·62	0·00209	291·880	1·11	1·17	0·620
T7B3	9980·0	48256·0	4·84	0·00187	317·816	1·21	1·42	0·630

local buckling occurred, the progression of relative movement between the inner and outer tubes would have been prevented, the integrity of the composite member would have been maintained, and the performance would have been enhanced. Therefore, it can be preliminarily suggested that tie rods should be placed where plastic hinges are likely to form.

5.4. Ductility

The ductility performance examined in this study concerned energy dissipated before member strength dropped to 80 per cent of the maximum strength of its corresponding CFT member. This criterion was set to avoid excessive p–Δ effects due to axial loading. Comparisons between strength and cumulative energy dissipated by test specimens are shown in Figure 8.

Flexural strength was calculated by multiplying the lateral force by the height of the loading point. As Table I shows, the maximum flexural strength of the T5 CFT member was 197 kN-m; therefore, the ductility is compared by the energy dissipated when members' strengths dropped to

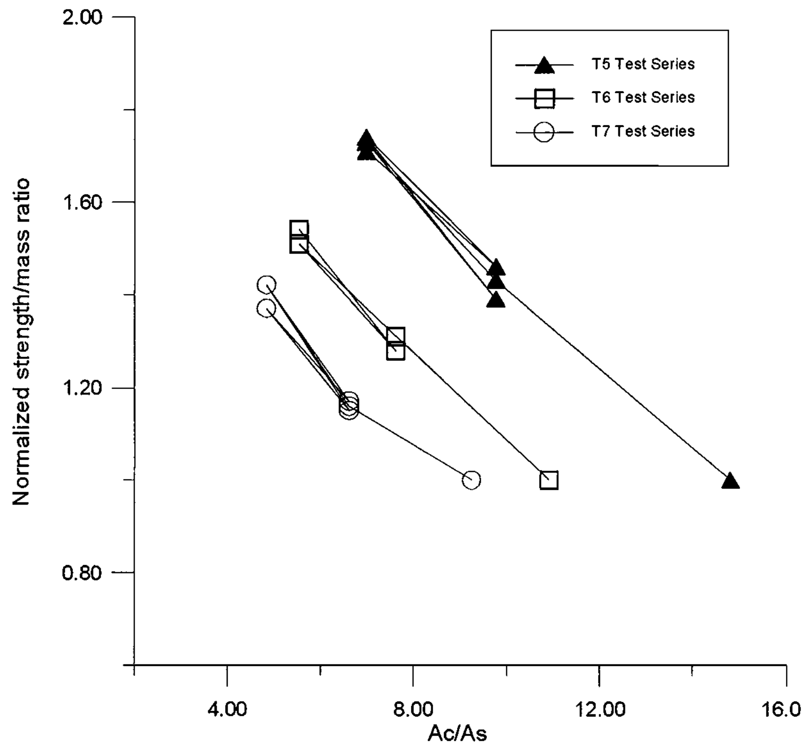

Figure 6. Relationship between A_c/A_s and normalized strength/mass ratios

156 kN-m (80 per cent of 197 kN m). According to this criterion, the T5B0 and T5S0 specimens dissipated 18 and 45 per cent more energy, respectively, than the T5 CFT member. The T6 series specimen with the larger inner tube (T6B0) dissipated 1·7 times the energy dissipated by the concrete-filled tube (T6). The specimen with the smaller inner tube (T6S0) also performed well, absorbing 50 per cent more energy than the corresponding CFT member (T6). In the T7 test series, the sandwich member T7B0 dissipated an amount of energy equivalent to that dissipated by the T7 CFT specimen; however, the T7S0 performed well, dissipating 1·4 times the energy dissipated by the T7 member.

By comparing the strength and energy dissipation capacities, it is found that the sandwich columns not only possess higher strength but also better energy dissipation capabilities than the CFT members. This phenomenon can be explained by the structure of the composites: the sandwich composite sections possessed higher flexural rigidity than the CFT member, and the inner tubes qualified as compact sections. As long as the inner tubes remained unbuckled, the tubes with integral in-filled concrete functioned as core supports for the outer tubes, thereby allowing the sandwich members to attain higher strengths and more cycles of deflections. The applicability of sandwich members to seismic design was further verified by the comparison of ductility enhancement, defined as the ratio of energy dissipated by sandwich members to that by the CFT member, shown in Figure 9.

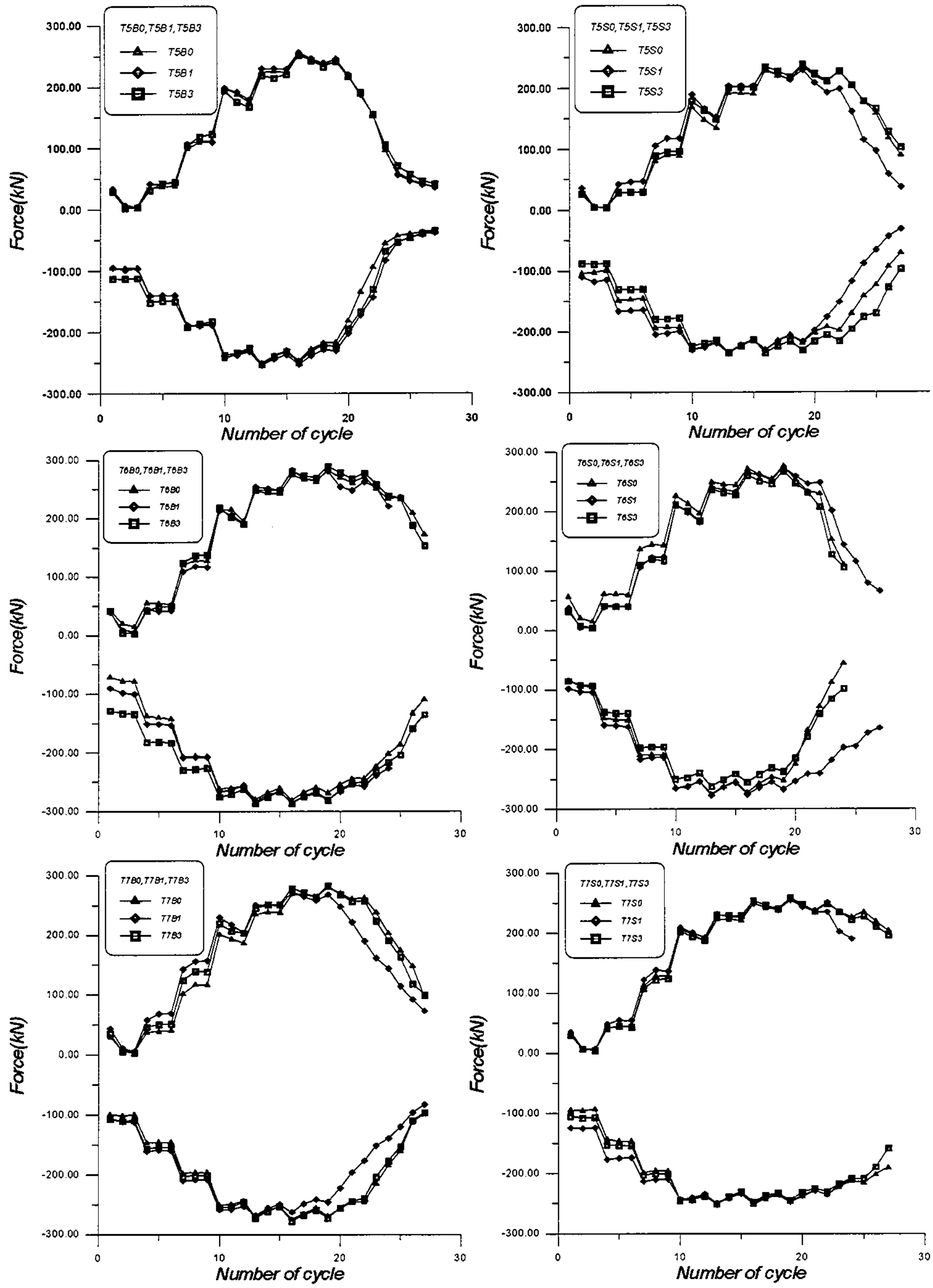

Figure 7. Comparison of member strengths for different numbers of tie rods

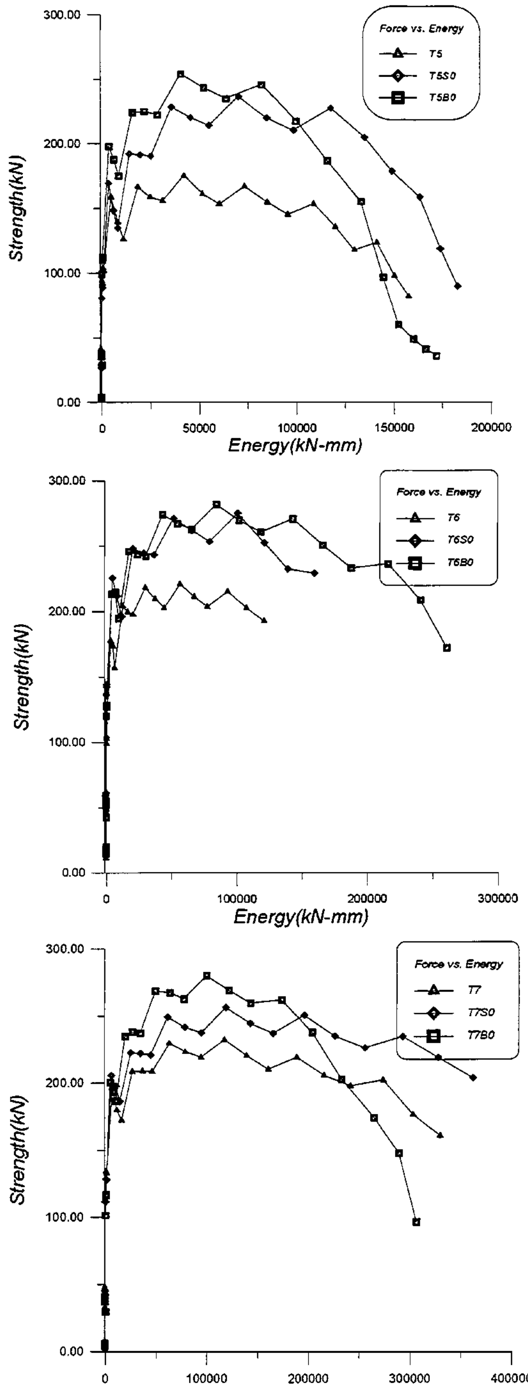

Figure 8. Relationship between strength and cumulative energy

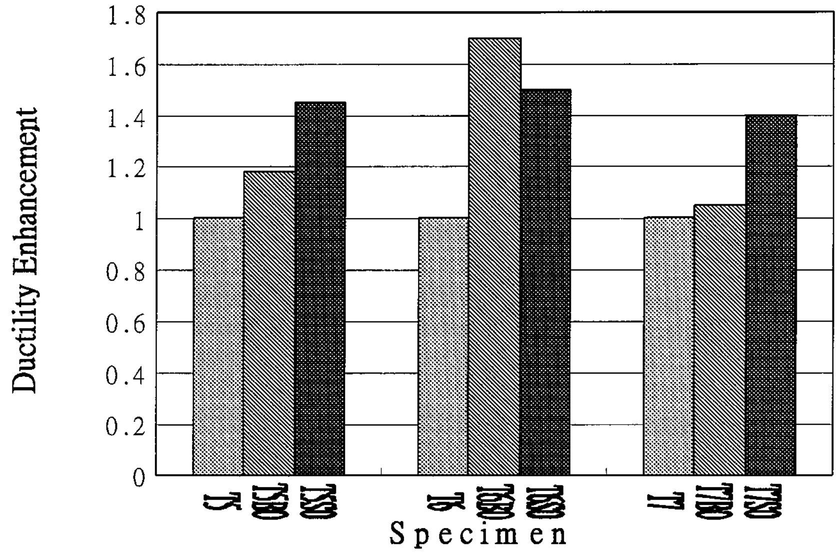

Figure 9. Ductility enhancement achieved in each test series

6. DESIGN APPROXIMATION

The applicability of sandwich box columns to engineering practice has been validated by their high-strength performance and energy-dissipation capacity as shown in the preceding sections. The following section discusses the flexural rigidity of sandwich box members in the elastic range, a most important service consideration to establish fundamental analytic information, because such members usually stay in the elastic range under normal conditions.

Since a sandwich box section is a composite formed by concrete and steel, its flexural rigidity depends on the interaction between its components. Theoretically, the neutral axis of the composite section must be defined in order to calculate the value. However, concrete is high in compression resistance and low in tensile strength; therefore, the neutral axis is not at the geometric centre. Furthermore, finding the genuine neutral axis of a composite member under seismic loading is difficult because the interaction between steel and concrete is not well-defined and the neutral axis is time-dependent during the application of loading. For simplicity and practical purposes, the flexural rigidity must be related to the member's geometric coordinate system. By approximating the contribution of concrete under compression as a fraction of the contribution of the gross concrete area, finding the flexural rigidity of the composite section (EI) can be expressed as follows:

$$(\mathrm{EI}) = (\mathrm{E_s I_s})_{\mathrm{C.G}} + \alpha(\mathrm{E_c I_g})_{\mathrm{C.G}} \tag{1}$$

where $(\mathrm{E_s I_s})_{\mathrm{C.G}}$ and $(\mathrm{E_c I_g})_{\mathrm{C.G}}$ are the flexural rigidities of steel tubes and the gross area of the uncracked concrete section with respect to the geometric centre, and α is a modifying coefficient accounting for the contribution of concrete.

The α values for all specimens are calculated and listed in Table I. Figure 10 shows the relationship between α and A_c/A_s ratios. The test results did not display a strong relationship

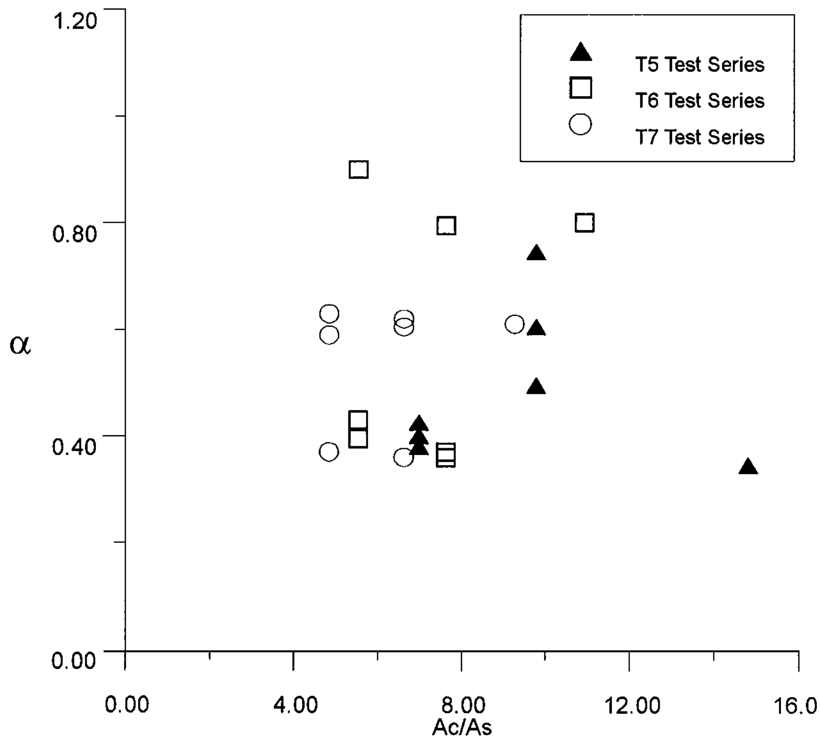

Figure 10. Relationship between α values and A_c/A_s ratios

between these two parameters; however, the comparisons did confirm the minimum stiffness achievable in sandwich members, either composed of compact or non-compact steel tubes. Therefore, a lower-bound approach in estimating stiffness of such members seems to be more practical. It is found from Figure 10 that all α values are greater than 0·345. For simplicity, a 0·35 value can be used for design purposes.

7. CONCLUSIONS

This paper discussed the beam-column behaviour of an efficient sandwich structural form modified from conventional concrete-filled tubes. Strength performance of the sandwich members was found to be higher than their corresponding CFT member. The improvement in strength reached up to 45 per cent for sandwich sections with non-compact outer tubes. For specimens with compact outer tubes, the maximum performance enhancement was 31 per cent, slightly lower than that of the group with non-compact outer tubes.

A fundamental conclusion can be drawn that the efficiency in strength improvement is higher for non-compact sections because they originally possessed lower capacities. When member mass is considered, the efficiencies of the structural forms were further enhanced. By examining the test results, it can be found that these efficiencies are proportional to their A_c/A_s ratios. Allowable

drifts for sandwich members were found to be higher than those for solid members. Energy-based ductility demonstrated that the sandwich members dissipated more energy than concrete-filled tubes under the same strength requirements.

Finally, a simplified expression for calculating elastic flexural rigidity was proposed. By examining the behaviour of members with larger and smaller inner tubes, it was found that when outer tubes were made of non-compact sections, an increase in inner tube size helped strength performance, although it was also disadvantageous in the post-buckling range. From the test results, a minimum 40 per cent ductility enhancement was confidently achieved in sandwich members with A_c/A_s ratios greater than 6·62.

Locations rather than the numbers of tie rods were the more dominant factors in affecting member behaviour. It is suggested that tie rods be placed at locations where plastic hinges are likely to develop so that performance can be further improved. The foregoing suggests sandwich members with lateral tie rods are valid potential design elements for use in earthquake-resistant engineering designs.

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REFERENCES

1. ACI, *Buildings Code Requirements for Structural Concrete* (*ACI 318–95*), American Concrete Institute, Detroit, MI, 1995.
2. P. F. Boyd, W. F. Cofer and D. I. Mclean, 'Seismic performance of steel-encased concrete columns under flexural loading', *ACI Struct. J.* **92**(3), 355–364 (1995).
3. Y. H. Chai, M. J. N. Priestley and F. Seible, 'Seismic retrofit of circular bridge column for flexural performance', *ACI Struct. J.* **88**(5), 572–584 (1991).
4. M. J. N. Priestley and R. Park, 'Strength and ductility of concrete bridge columns under seismic loading', *ACI Struct. J.* **84**(1), 61–76 (1987).
5. A. W. Taylor, R. B. Rowell and J. E. Breen, 'Behavior of thin-walled concrete box piers', *ACI Struct. J.* **92**(3), 319–333 (1995).
6. T. Usami and H. Ge, 'Ductility of concrete-filled steel box columns under cyclic loading', *J. Struct. Engng.* **120**(7), 2021–2040 (1994).
7. S. Wei, S. T. Mau, C. Vipulanandan and S. K. Mantrala, 'Performance of new sandwich tube under axial loading', *J. Struct. Engng.*, **121**(12), 1806–1821 (1995).
8. AISC, *Load and Resistance Factor Design Specification for Structural Steel Design*, American Institute of Steel Construction, Chicago, IL, 1994.

